Dynamic Investigation of an Ancient Masonry Bell Tower with Operational Modal Analysis

A Non-Destructive Experimental Technique to Obtain the Dynamic Characteristics of a Structure

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Abstract: This paper shows the results of an experimental analysis on the bell tower of "Chiesa della Maddalena" (Mola di Bari, Italy), to better understand the structural behavior of slender masonry structures. The research aims to calibrate a numerical model by means of the Operational Modal Analysis (OMA) method. In this way realistic conclusions about the dynamic behavior of the structure are obtained. The choice of using an OMA derives from the necessity to know the modal parameters of a structure with a non-destructive testing, especially in case of cultural-historical value structures. Therefore by means of an easy and accurate process, it is possible to acquire in-situ environmental vibrations. The data collected are very important to estimate the mode shapes, the natural frequencies and the damping ratios of the structure.

To analyze the data obtained from the monitoring, the Peak Picking method has been applied to the Fast Fourier Transforms (FFT) of the signals in order to identify the values of the effective natural frequencies and damping factors of the structure. The main frequencies and the damping ratios have been determined from measurements at some relevant locations. The responses have been then extrapolated and extended to the entire tower through a 3-D Finite Element Model. In this way, knowing the modes of vibration, it has been possible to understand the overall dynamic behavior of the structure.

Keywords: Bell tower, Dynamic behavior, Environmental vibration, Masonry structure, OMA.

1. INTRODUCTION

The paper deals with the analysis of specific problems common to slender structures, "(such as towers and belfries)" in order to understand the structural behavior and the related crack patterns. In recent years, in fact, their particular geometrical characteristics have been a topic discussed in many research focused on the evaluation methods to discover the dynamic behavior of these structures in masonry [1], concrete [2-4] and reinforced concrete [5].

Among the buildings belonging to the heritage of Italian architecture, masonry structures characterized by a predominantly vertical development, such as towers and belfries, are really widespread. The low ductility of masonry materials, associated with the thinness of these structures similar to a cantilever, can usually lead to a quite brittle global behavior; therefore these structures are particularly vulnerable to dynamic actions. The preservation of our architectural heritage from the seismic action is of prominent concern in our country, due to recent seismic events and the richness of our architectural heritage, which consisting primarily of masonry buildings not originally designed to resist to horizontal actions. In the past, in fact, the construction concept was focused mainly on the action of vertical loads and not on the strength and ductility to horizontal actions. This explains the inability of masonry buildings to resist tensile stresses and the proposal from some scientists of a unified treatment of masonry structures presenting an overall and comprehensive up-to-date based on the No-Tension (NT) Theory [6-9].

The research aims, through modal analysis, to calibrate a numerical model on the real behavior of the bell tower of "Chiesa della Maddalena" (Mola di Bari, Italy) and to have the possibility to predict the change in the global behavior of the tower after a static consolidation work. Operational Modal Analysis (OMA) has been applied for the dynamic identification of the tower. The choice of using an Operational Modal Analysis method derives from the necessity to know the modal parameters of a structure with a non-destructive testing because, as in this case, the structure has a cultural-historical value. The technique allows the possibility to extract the modal parameters (natural frequencies, mode shapes and damping ratios) from output-only experimental data obtained by mean of ambient vibration testing [9, 10]. And a Finite Element Analysis (FEA) can give the possibility to correlate the numerical results with the experimental ones.

The actual boundary conditions of the tower of "Chiesa della Maddalena" and the mechanical properties of masonry are defined through the dynamic identification based on the recordings of an experimental sampling in-situ, which included the use of only ambient vibrations. This represents,

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Fig. (1). Bell tower of "Chiesa della Maddalena": North side (**a-b**), South Side (**c**).

through input of environmental nature, a solution to evaluate the characteristics of the materials and the constraint conditions of the structure. Then, by mean of updating procedures the aim is to define reliable numerical models to validate on the real data the vibration modes, the natural frequencies and the damping ratios of the structure.

The same techniques will be utilized to monitor two ancient towers in the framework of Interreg SMART BUILT project (project founded under the European Cooperation Programme Greece-Italy 2007-2013 "Investing in our future").

One main problem in the Operational Modal Analysis (OMA) is the maintenance of the data acquisition system [11]. Field measurements, in fact, are often performed in very harsh environmental conditions, requiring high accuracy both in the control of the test set-up and in the analysis of measured data. Moreover, measurements are usually noisy for the necessity of using very long cabled transducers. Other relevant and common problems in the field of OMA are [12]: the difficulty to detect different mode shapes for very closely spaced modes, the subjectivity in the system order estimate, the need of a more efficient algorithm able to omit the spurious modes created by noise or redundant degrees of freedom of the model.

Furthermore, this type of investigation is particularly suitable to slender structures, as explained in the Italian guidelines [13], because if they are subjected to vibrations even of low intensity, generally very clear signals are produced. Sometimes, on the contrary, difficulties arise due to the geometry and the accessibility of old masonry towers and buildings.

In the present case, in fact, the tower is connected to the church on three sides. In particular, the West and the North masonry walls of the tower are totally incorporated into the walls of the church. Also the positioning of the accelerometers was difficult due to the inability to place at a higher level, it was impossible to get to the top due to lack of a staircase. As a consequence the accelerometers were too close to the floor connected to the church. This could produce some differences from the real values of the mechanical parameters of the tower.

In parallel to the dynamic identification by a 3D finite element model, linear simplified analysis was carried out. This was a static and kinematic limit analysis with convenient constitutive equations to describe the response of the masonry tower, which was approximated with a simplified model as a cantilever with all the degrees of freedom restrained at the base. In that analysis, the mechanical behavior of the material was considered to be linear elastic, the method of the superposition was applied and the weight of the material was supposed to be constant for the whole bell tower.

2. DESCRIPTION OF THE TOWER

Originally the Church was a chapel, under the name of "Santa Maria". There was only one altar with an ancient venerated image of the "Penitente Maddalena". It was located in "Mola di Bari" and was built at the wish of Charles I of Anjou on the way to Rutiliano, a nearly town outside the city walls of that time, as explained by an ordinance dated September 22, 1279 "…extra fortelliam ipsius terre ex parte pred. terra Rutiliani".

Between the end of the XV century and the beginning of the XVI century the chapel was repeatedly threatened to be destroyed, due to the different war events that took place in those years in the territory of "Mola di Bari". With the urban sprawl of the new borough around the chapel was built a church which could sustain the increasing population and satisfy religious duties.

The works for the construction of the new church, "Chiesa della Maddalena", began in 1617. The church and the bell tower were built in parallel, but at the end of the XVII century the tower was almost entirely destroyed by lightning, it was rebuilt between the end of the XVII and the beginning of the XVIII centuries. These historical sources do not specify which parts of the tower were destroyed, or if the materials were reused. Certainly the type of material and the technique employed were the same, indeed there are no visible differences between the base and the soaring structure.

The Church and its belfry, over the years, underwent several architectural restorations in the XVIII century and again in the XIX century. The only restoration of the bell tower, which is clearly documented, was performed to in 1905. The drive system of the bells was replaced and upgraded, adding the hammers connected to power motors. Still it is possible to notice the missing parts in the wall, probably due to the old manual functioning to ring the bells. As is easily deduced access to the bell cell has been modified over the time, but there are not historical sources to this regard.

2.1. Geometrical Description and Analysis Of Damages

The bell tower Fig. (**1a**, **b** and **c**) is built entirely with calcareous tufa, a stone typical of the Apulia region, which should not be confused with tuff, a porous volcanic rock. In fact tufa is a variety of limestone, formed by the precipitation of carbonate minerals from ambient temperature water bodies. It has a white, yellow or red coloration and particular properties like at a high porosity, lightness and toughness. The tufa has high technical characteristics and in particular a good compressive strength.

The tower has a 4.38x4.11 m rectangular plan Fig. (**2**) and it is 34.7 m tall. More in detail the height of the basement is 18.30 m, the central body is 11.50 m tall and the height of the dome is 4.90 m Figs. (**3a**, **b**, and **c**). The tower is divided into three parts, separated by two broad cornices: the base, the central body - where there are two rows of windows -, and the top where the Russian dome is placed. Inside, the tower appears to be almost completely empty Fig. (**4**). In fact, there is only a barrel vault, necessary in a bell chamber, which can be accessed through the internal staircase.

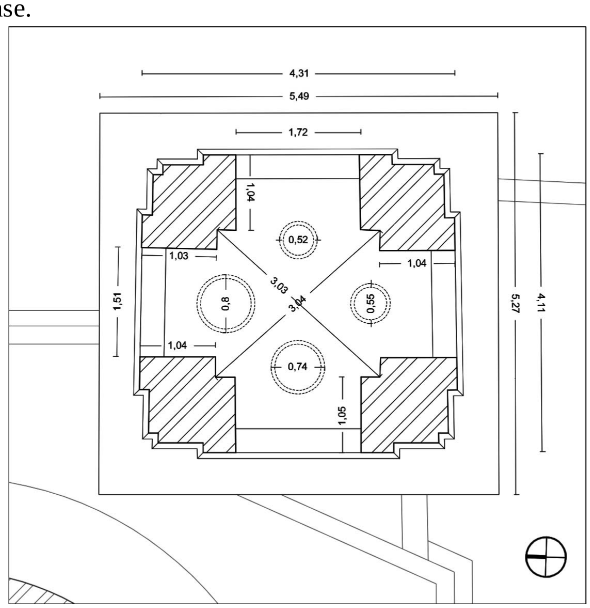


Fig. (2). Plan of the Bell tower.

In the central part there are two rows of windows in composite style, like the entire structure; the dome has also two openings, one on the North side and the other on the South side.

The bell tower is joined to the church for 17.9 m on the North side; for almost the entire West wall and on the South façade it is joined to an adjacent chapel, which is 11.8 m tall.

There is no significant damage, but only minor cracks on the keystones of the openings (which can be easily compensated in a future restoration) and two cracks on the North side and on the South façade both below the openings; maybe during the restoration work done in the XX century they were compensated and, until now, it is important to emphasize they have not reopened. These cracks do not start from the bottom, so they are certainly not attributable to a failure of the foundations, but to the complex and variable stiffness of the structure.

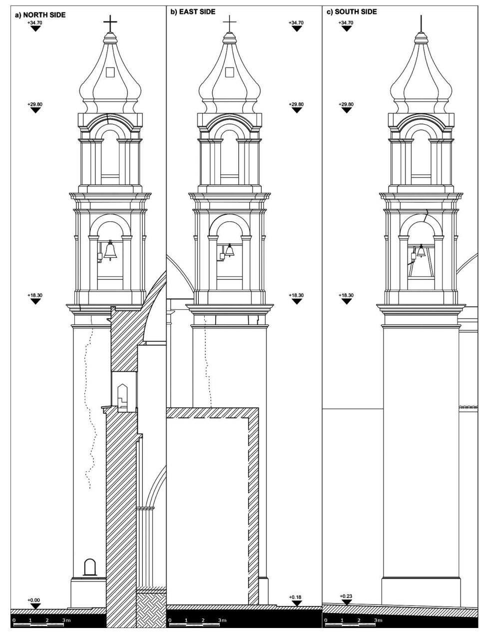


Fig. (3). Prospect North (**a**), Prospect South (**b**), Prospect East (**c**).

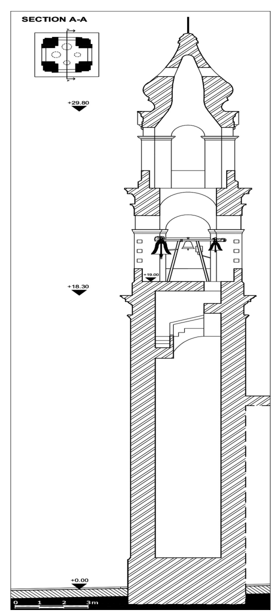


Fig. (4). Section North-South.

The four bells are housed under the arches (one on each side of the tower) and are connected and linked to the structure through steel profiles, which discharge their weight on the masonry, on to which they were first clamped and then blocked through concrete mix.

Fig. (5). Accelerometer n°2 (**a**), Accelerometer n°4 (**b**), Instrumental system (**c**).

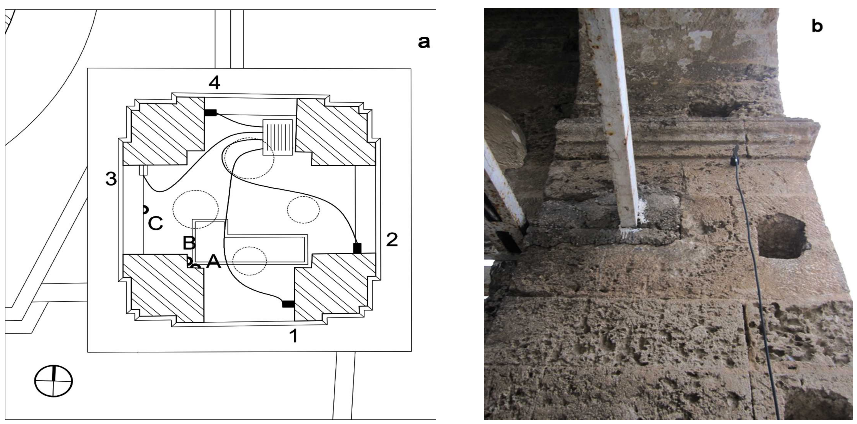


Fig. (6). Positioning scheme of the accelerometers (**a**), Accelerometer n°1 (**b**).

3. BELL TOWER EXPERIMENTAL RESPONSE

3.1. Instrumental and Data Acquisition System

The vibration data collecting is based on the use of a portable compact platform equipped with a personal computer. The chain of acquisition is composed of:

1. Four piezoelectric accelerometers, with a working range between 0.5 and 2000Hz and a voltage sensitivity, mV/g for both the last two accelerometers Fig. (**5a** and **b**);
2. Co-axial cables with low impedance and with a length variable from 4.0 m to 15.0 m;
3. Multi-channel acquisition system (National Instruments-NI 9234), with four simultaneously sampled analog inputs with a ±5 V input range. It has a maximum sampling rate of 200 kHz pre-channel, 24 bit resolution, 102 dB dynamic range and anti-aliasing filters Fig. (**5c**);
4. Platform NI Compact DAQ DSA, with eight slots: in each one it is possible to integrate a NI 9234, having the same characteristics.

The management of the acquisition and archiving of the data is done by means of a software developed in a Lab view ambient [14]. The program automatically converts electric units to engineering units, and stores the signals acquired on the Hard Disk.

The signals post-processing system, an FFT Properties-Signal Analyzer [15], has been chosen in order to obtain characteristics of precision during the signals acquisition.

3.2. Experimental frequencies

To perform the tests the accelerometers have been positioned in the bell cell, the latter being the only height reached without any special equipment. The data have been acquired in two horizontal directions with regard to the N-S and E-W principal axes of the tower. A sampling frequency of 1653 Hz has been utilized.

In Fig. (**6a**) the positioning of the four piezoelectric accelerometers is shown. In particular, they were arranged at a height of 2.90 m from the extrados of the vault (18.30 m), due to the impossibility to reach a higher quota, and 0.30 m from the external edges of the tower Fig. (**6b**).

The bells, even if still in place, were not utilized as a forced input during the tests as they are not installed directly on the masonry walls of the chamber bell, but through a metallic frame, independent from the structure of the bell tower. Therefore the acquisition data are obtained using only two different sources: the ambient noise (considering different factors such as wind, traffic, etc.) and the impulse on the bell cell using a hammer. In particular, for the last type of excitation two different positions shown in Fig. (**6a**) have been defined.

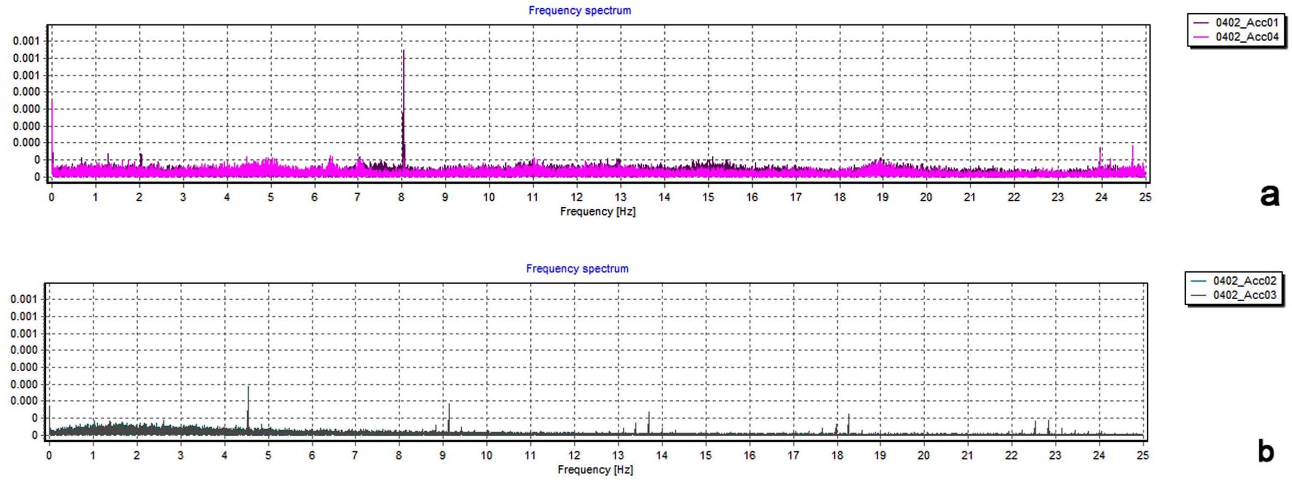


Fig. (7). Power spectrum in the N-S direction (**a**) and the E-W direction (**b**) with frequencies in the range [0-25Hz].

Table 1. First three natural frequencies of the bell tower

Accelerometer	Direction	Frequency (Hz) 1° ambient acquisition			Frequency (Hz) 2° ambient acquisition			Frequency (Hz) 3° ambient acquisition		
2 - 3	N - S	4.57	9.15	13.7	4.57	9.15	13.7	4.57	9.15	13.7
1 - 4	E - W	8.06	23.9	-	8.05	23.9	-	2.1	7	12.1

The data acquisition tests took place on the 2nd of February 2012, from 1.00 p.m. to 17.30 p.m. on a particularly windy day.

The tests involved the misure of vibrations for three series of acquisitions of 15 min, lasting 45 min in total. The vibrations were induced by the wind and the passage of motor vehicles, registered by the sensors positioned on the bell tower in the two perpendicular horizontal directions. Many of the signals measured by the accelerometers were "parasites"; therefore they have been identified and filtered by the software at a frequency of 50 Hz.

The acquisitions made using the hammer lasted 2 min. For each acquisition the structure was struck three times, and precisely on:

A. the S-W pillar in the N-S direction;

B. the S-W pillar in the E-W direction;

C. the West parapet in the E-W direction.

From the recordings, using a Fast Fourier Transform (FFT), the frequencies have been obtained together with the power spectral response of the structure in the N-S and E-W directions Fig. (**7**).

The results of the tests with the hammer were not considered for the present study because the strokes were totally absorbed by the masonry of the adjacent church.

Table **1** shows the significant frequencies obtained, that is the peak at 8.05 Hz in the E-W direction (T = 0.1227 s) and at 4.57 Hz in the N-S direction (T = 0.2188 s).

The frequencies determined in the last acquisition for accelerometers 1 and 4 have provided very different results, due to the presence of some noise; therefore they have been neglected in the analysis.

Once a natural frequency has been detected, the logarithmic viscous damping decrement is easily derived from the measured time history [16]. For this tower an approximate damping ratio of 0.01745 was obtained.

4. NUMERICAL ANALYSES

The preliminary analysis developed is simply numerical. The formulas suggested by UNI EN 771-6:2011 have been applied to calculate the natural frequencies of the structure modeled as a simple cantilever. Taking into account the presence of the adjacent church, two models were assumed:

1) The tower was considered standing free for its entire height;

2) The length of the cantilever model is equal to the actual free part of the tower.

Clearly the second model is closer to the real one, which takes into account the existing boundary conditions along the height of the tower. The results obtained with the latter model are very close to those acquired in-situ.

4.1. 3D Finite Element Model

In order to obtain more accurate results, a 3D finite element model (FEM) was developed based on the geometrical survey, even if some simplifications have been applied. So, the different constraint conditions that characterize the structure on the North, South and West sides have been considered as local elastic springs. The tower has been modeled with 4-node tetrahedral solid elements with a uniform mass distribution along the structure.

Table 2. The final frequency values of the F.E.M

Mode	Frequency (rad/s)	Frequency (Hz)	Mode classification
1	28.02	4.46	1st Bending Mode E-W
2	28.76	4.58	1st Bending Mode N-S
3	76.41	12.16	1st Torsion mode
4	104.57	16.64	2nd Bending Mode E-W
5	110.13	17.53	2nd Bending Mode N-S
6	124.65	19.84	1st Sussultatory mode
7	147.18	23.43	1st Diverted bending mode
8	160.72	25.58	2nd Diverted bending mode

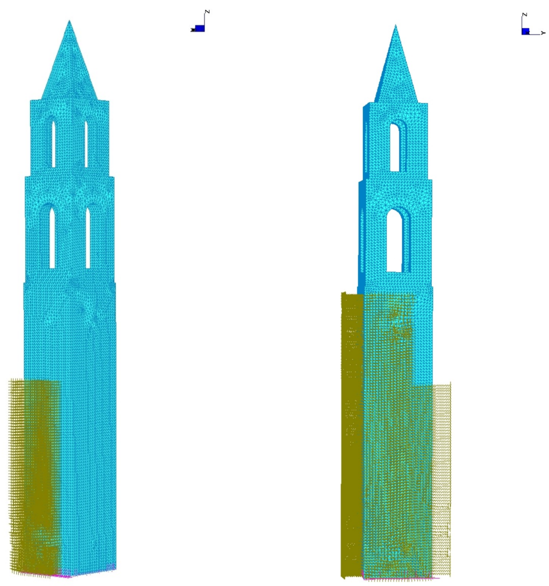


Fig. (8). South-East façade (**a**) and West façade (**b**) of the Finite Element Model.

Figs. (**8a** and **b**) show the final model, generated using Strauss 7 [17], consisting of 132 vertices, 54103 node and 24265 brick elements.

The frequencies obtained by the in-situ acquisitions are so different in the two directions that they cannot both be related to the first bending mode. In fact, for an almost square plan, the two first bending modes should be close. Therefore the frequency of 8.05 Hz will be a second flexural mode in E-W direction, because the value is excessively high and too different from the frequency of 4.57 Hz in the N-S direction. The Italian code [14] forbids calibrating the model on the higher modes, because at higher frequencies the non-linear behavior of the structure involves too many parameters, which are difficult to handle with a model. As a consequence, the value of 8.05 Hz has not been taken into account, even if it was measured in situ.

The final masonry mechanical properties used for FEM analysis are the elastic modulus (E) equal to 7500 MPa, the Poisson's modulus (ν) equal to 0.15 and the material density equal to 1700 Kg/m^3. It was assumed a linear elastic mechanical behavior during the calibration. The value of the stiffness of the springs (K) is equal to 4500 N/mm. These values were updated following an iterative approach until a satisfactory level of agreement between numerical and experimental frequencies was obtained. In this particular case the iterative process was carried out varying the Young's modulus (E) of the masonry in order to get values very close to the real bending and torsional frequencies obtained from the in-situ tests. After this calibration, for a more accurate approximation, the value K of the springs has been varied to get closer to the experimental behavior. The springs are applied in the lower part of the tower, along the sides in contact with the Church. In this way it has been possible to get the response of the natural frequencies more similar to the values obtained from the experimental acquisitions. Moreover, from these results it is possible to affirm that the 3D model is always the model closest to the real situation.

The data of the natural frequencies obtained from Strauss 7 are shown in the Table **2** together with the mode classifica-

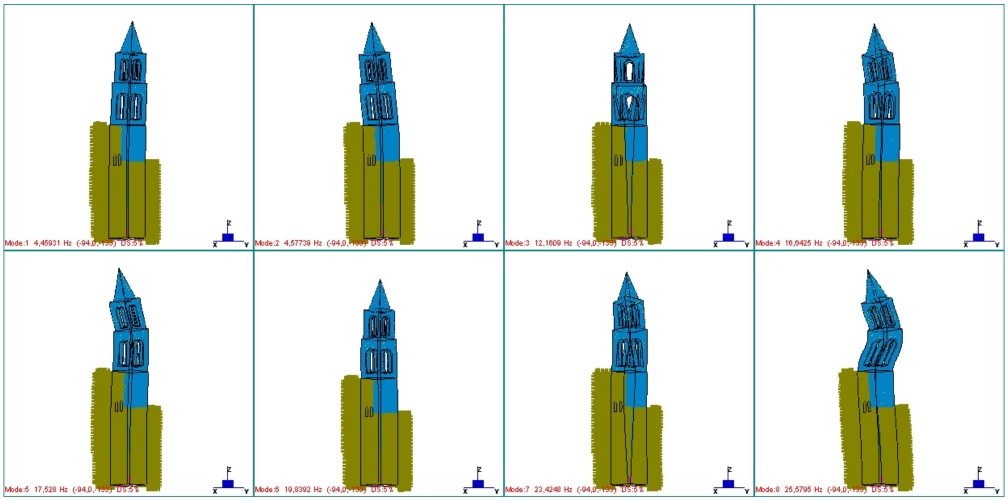
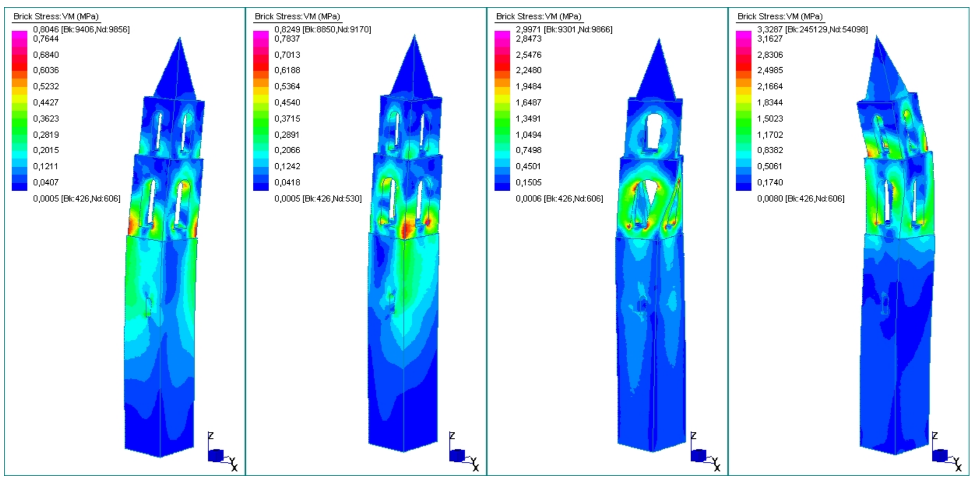
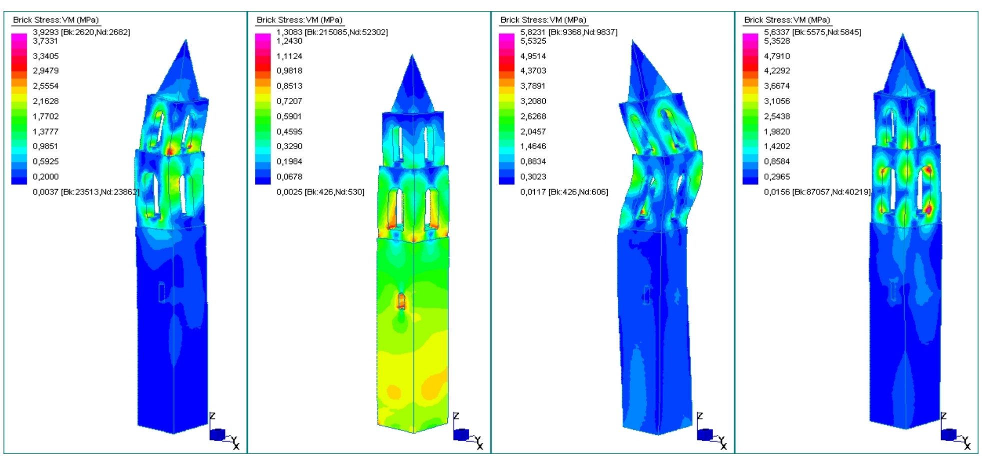


Fig. (9). Modal shapes of vibrations of the Finite element model.



a



b

Fig. (10(a-b)). Stresses of the vibration modes from the Finite element model.

tion. The correlated modal shapes of vibration are shown in Fig. (**9**).

The first mode corresponds to a bending mode in E-W direction, the second to a bending mode in N-S direction and the third mode is a torsional one.

As expected, the first two bending modes are very close having the structure an almost square plan. This bell tower has a first natural frequency higher than in any other similar cases presented in scientific literature [1,16,18]. This is a singular aspect for this type of tower that may be explained by its high stiffness due to the presence of the constraints with the next building of the Church.

Through the model and the analysis of the natural frequencies it is possible to estimate the stress, strains and displacements of the tower. In particular it is useful to understand the stress distribution in the masonry in Figs. (**10a** and **b**) with the N-W angulation.

CONCLUSIONS

The bell tower of "Chiesa della Maddalena" is a really particular and difficult case for monitoring because of its extreme rigidity due to the boundary conditions especially on its West and North sides where it is connected to the adjacent church walls, and the impossibility to place the accelerometers at higher levels on the walls. For these reasons frequencies higher than those of similar masonry structures (2,5-3 Hz) have been obtained.

The 3D FE model results more accurate than the preliminary analysis based on the code prescriptions because it considers the real mass distribution of the structure, including the voids. Therefore, the 3D FEM has been useful to perform the analysis of the stresses, which allows to better understand that the most stressed part of the structure is the level in correspondence to the bell cell, both at bending and torsional modes, being that part of the structure also characterized by masonry pillars, which are elements of weakness.

CONFLICT OF INTEREST

The authors confirm that this article content has no conflicts of interest.

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